Initial Drainage Report
for
Hot Weather Test Complex
at
US Army Yuma Proving Ground

Prepared for:

US Army Yuma Proving Ground
Roadrunner Drop Zone Relocation
HWTC Stormwater Assessment
Task I.D. 01-065-CB-9F27

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Table of Contents

List of Figures

List of Tables

Appendices

1 INTRODUCTION

1.1 Background and Scope

Premier Engineering Corporation (Premier) was contracted by Jason Associates Corporation (Jason) to perform a initial drainage analysis at the site of the proposed Hot Weather Test Complex (HWTC). The site for the proposed HWTC is located within the Laguna Region of the US Army Yuma Proving Ground (YPG), about 25 miles northeast of Yuma, Arizona. Figure 1 shows the location of the proposed HWTC at YPG.

The proposed test track and facilities will cross several drainage ways; therefore, as part of the design, off-site drainage concerns and ways of passing these flows must be addressed. Additionally, on-site flows generated by the test track and facilities will have to removed and routed to appropriate facilities meeting both vehicular safety and environmental concerns. The purposes of this report are:

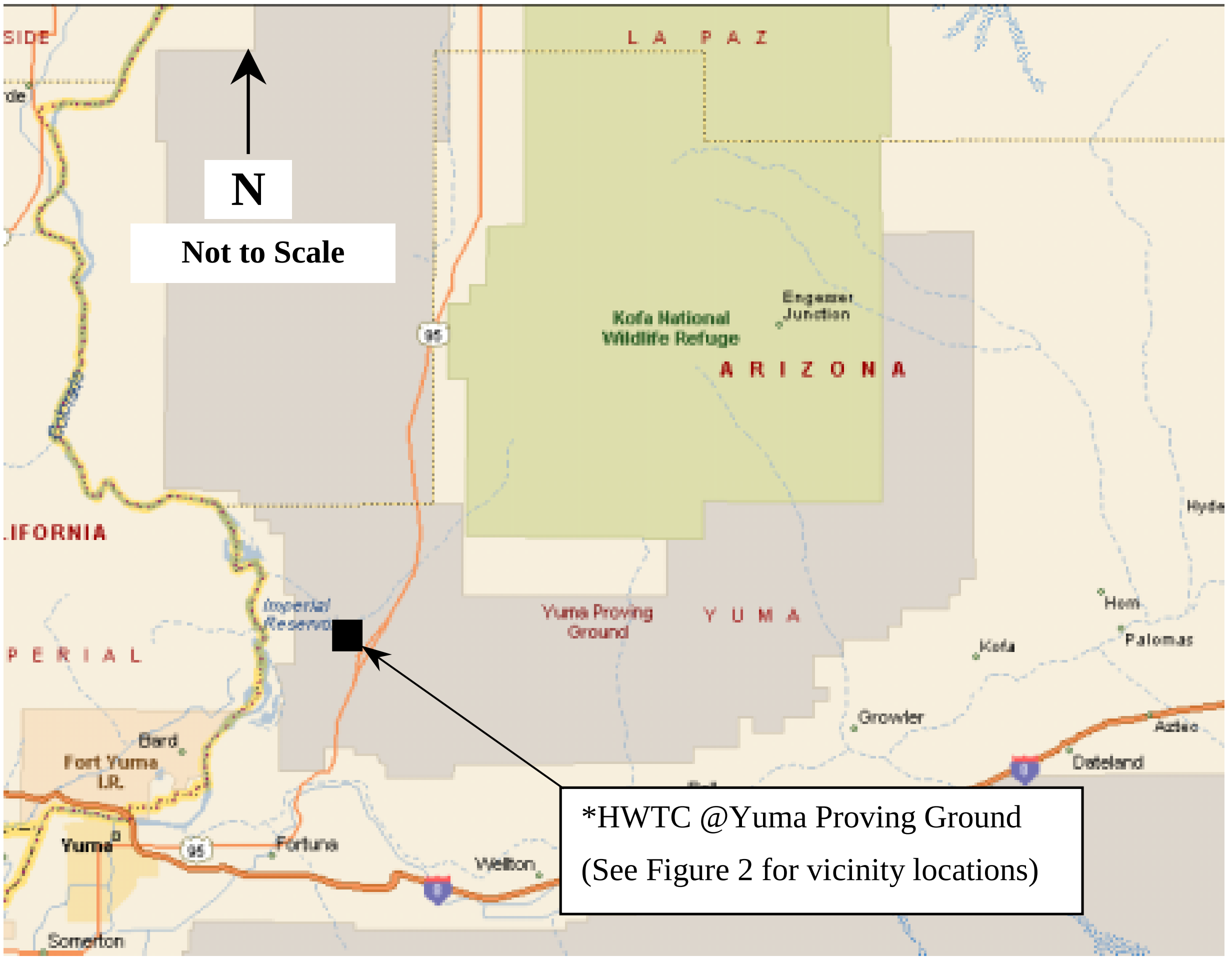
1. Determination of the off-site peak flow rates impacting the site.
2. Evaluation of how flows are to be conveyed through the site.
3. Preliminary determination of the required design features to deal with the drainage conditions identified at the site.

1.2 Previous Reports and Studies

There were no identifiable previous drainage reports for this project.

1.3 Floodplain Considerations

Since the subject site is located on a military reservation, there are no Federal Emergency Management Agency (FEMA) floodplain delineations for this site.


N
Not to Scale
L A P A Z
Kofa National
Wildlife Refuge
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Junction
A R I Z O N A
IFORNIA
Imperial
Yuma Proving
Ground
Y U M A
Hyde
Horn
Palomas
Kofa
Growler
Aztec
Dateland
Bard
Fort Yuma
I.R.
Yuma
Fortuna
Wellton
Somerton
*HWTC @Yuma Proving Ground
(See Figure 2 for vicinity locations)

Figure 1

Figure 2: ***Vicinity Map***

(Insert here)

2 EXISTING CONDITION

2.1 Topography

The proposed HWTC site is situated on the Castle Dome Plain with a contributing drainage area flowing in a southwesterly direction. Formation of the plain is a result of deposition from weathered soils washed down from the surrounding hills to the north. The high point of this watershed is approximately 780 feet Mean Sea Level Elevation (MSLE). The contributing watershed is typical of low, southwest desert regions. Surrounding hills are well defined, with steep drainage ways (greater than 10%), natural rocky, incised channels, and little vegetative cover. These channels carry weathered sediments to the desert floor, where they fan out and create flat, deep alluvial deposits containing slopes of approximately 1-5 percent. The project site is located at an elevation of about 500 feet MSLE.

Vegetation is typical of the arid Sonoran desert region. Creosote bush and white bursage are the dominant plant species located throughout the plain. In and around the washes are ironwood, yellow paloverde, various understory shrubs, and an occasional ocotillo.

2.2 Existing Condition

2.2.1 Legal Boundaries

The proposed area for the HWTC is completely within YPG boundaries. The site is located in Yuma County, Range 19W T16S, and is bordered to the east by US Highway 95.

2.2.2 Access Control

The US Army controls vehicular access to the HWTC site. Gravel roads provide access from the site to US Highway 95, which is located approximately 0.25 miles to the east.

2.2.3 Land Use

The hydrologic land use type within the area is termed desert rangeland with mountainous features in the upper portions of the watershed. Note that this is a hydrologic category and does not reflect actual land use. The site in question is used strictly for test and evaluation of military materiel and equipment.

2.2.4 Buildings and Other Structures

There is a portable radar tracking station and an observation hill, towards the center of the proposed test track location.

2.2.5 Drainage and Flood Control Barriers

One check dam is denoted at the upper portion of the watershed on the USGS 7.5 minute quadrangle map. It was apparently built to ensure that stormwater generated to the north is passed to the east of US Highway 95 and not south through the study area. This assumption will be verified by a site visit at a later date. Numerous dirt roads crisscross the study area, primarily at grade, having minimal conveyance potential. Additionally, it did not appear that these roads would block or divert significant flows.

3 HYDROLOGY

Peak flow rates were calculated for design and planning purposes. Synthetic storm/runoff models were developed for the site using the Army Corps of Engineers' (COE) program HEC-1 (Version 4.1). The following sections provide in depth discussion on the preparation of the models. The methods and procedures used to model the watershed are found in the Flood Control District of Maricopa County (FCDMC) *Drainage Design Manual for Maricopa County, Arizona, Volume I Hydrology*. Calculations were performed in English units.

3.1 Drainage Basin Delineation and Topography

Drainage basin information, such as basin boundary and flow path alignments, lengths and slopes, were obtained from the following sources:

- U. S. Geological Survey 1:24,000 scale topographic maps with 20-foot contour interval, Middle Mountain South, Middle Mountain North, and Imperial Reservoir quadrangle maps.
- Field observations (Photos)

Total drainage area was broken into 24 contributing sub basins. Sub basin delineation was based upon topographic and land use homogeneity and size. The following assumption was made affecting the delineation of the watershed. At the check dam it was assumed that all off-site flows routed east under US Highway 95 would flow into Castle Dome Wash. This assumption was based upon aerial photos and will have to be confirmed by further field inspection. Figure 3 shows the watershed delineations.

3.2 Precipitation

Rainfall data for this report was determined using the values found in the FCDMC manual. Since the watershed was less than 20 square miles, the 6-hour duration storm was used. The frequencies of storm events modeled were the 5, 10, 25, and 100-year events. Precipitation point rainfall values were found using the isopluvial (rainfall) maps located in the Arizona Department of Transportation (ADOT) Hydrology manual. Precipitation data can be found in Appendix A.

Figure 3: ***Watershed Delineation***

3.3 Rainfall Losses

Rainfall loss data for this report was determined using the Green Ampt-Method. Green-Ampt rainfall loss parameters were calculated based upon soil data from *Yuma Proving Ground, Soil Survey 1991.* Five soil associations were found in the watershed.

1. **No. 1. :Riverbend family- Carrizo family complex** (1 to 3 percent) – 47 percent Riverbend, 41 percent Carrizo, 12 percent inclusions. Consists of gravelly/cobbly sandy loams, gravelly coarse sand, and gravelly loamy sand.
2. **No. 2. :Cristobal family-Gunsight family**, gypsiferous substratum complex (1 to 15 percent slopes) – 59 percent Cristobal, 30 percent Gunsight, and 11 percent contrasting inclusions. Consists of extremely gravelly silt loam and extremely gravelly sandy loam.
3. **No. 4.: Gunsight family- Chuckawalla family complex**, gypsiferous substratum, (5 to 45 percent slopes)- 65 percent Gunsight, 24 percent Chuckawalla, and 11 percent contrasting inclusions. First 5 inches consists primarily of very gravelly fine sandy.
4. **No. 5.: Superstition family Rositas family complex,** (1 to 15 percent slopes) 56 percent Superstition, 28 percent Rositas and 16 percent contrasting inclusions. First 60 inches is loamy sand and sand.
5. **No. 9. : Lithic Torrirthents and Typic Torriorthents soil,** (15 percent to 60 percent) rocky – 47 percent Lithic Torriothents, 32 percent Typic Torriorthents, and 6 percent rock outcrop. First 8 inches consists primarily of very gravelly fine sandy loam and rock outcrop.

Soil delineations are shown in Figure 4. This figure reflects soil associations and concentration points for each sub-basin. Green-Ampt parameters for the various soil textures were derived using Tables 4.1 and 4.2, and Figures 4.3 and 4.4, of the FCDMC Volume 1 drainage manual. Rainfall loss data utilized for the project is shown in Appendix C.

3.4 Unit Hydrographs

The FCDMC manual suggests using the Clark Unit Hydrograph method for basins less than 5 square miles. Therefore, the larger basin (greater than 160 acres) outflow hydrographs were calculated using the Clark Unit Hydrograph method. This method requires the estimation of three parameters the time of concentration (T_c), the storage coefficient (R), and a time-area relation (See HEC-1 UA card as found in the appendix for FCDMC excerpts pertaining to this topic.)

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The time of concentration is the travel time, during the corresponding period of most intense rainfall excess, for a floodwave to travel from the hydraulically most distant point in the watershed to the point of interest. The Papadakis-Kazan Equatin used is:

$$\mathbf{Tc = 11.4{*}L^{0.50}{*}K_b^{0.52}{*}S^{-0.31}{*}i^{-0.38}}$$

The FCDMC's Rational Method uses the same equation to determine Tc. A more in-depth explanation/description is presented in section 3.5.1.

The storage coefficient is a Clark Unit hydrograph parameter that relates the effects of direct runoff storage in the watershed to unit hydrograph shape. The equation for estimating the storage coefficient (R) is:

$$\mathbf{R = 0.37\ Tc^{1.11}\ L^{0.80}\ A^{-0.57}}$$

Where R is in hours and the variables are as defined for the T_c equations.

The time-area relation is a graphical parameter that specifies the accumulated area of the watershed that is contributing runoff to the outlet of the watershed at any time. The hydrologic land-use in the watershed, or sub-basin, is desert/rangeland or is mostly desert/rangeland with some mountains/hills in the watershed. Based on this hydrologic land use, the synthetic time-area relation for the Clark unit hydrograph utilized for this project should be best represented as Natural Watershed. Refer to Appendix D for Clark unit hydrograph input for each sub-basin. Note, that the Maricopa County Unit Hydrograph Procedure1 (MCUHP1) program calculates the Tc for each basin and storm event (e.g. the 5, 10-year storms). A summary of the input used to determine Tc and R is presented in Table 1. Figure 4 shows the soils disposition at the project site. Appendix C contains the values and calculations pertaining to this site.

Figure 4: ***Study Area Soils Map***

Table 1: Tc & R Input Parameters

3.5 Rational Method

For basins less than or equal to 160 acres, FCDMC Rational Method was used to estimate peak flows. The Rational Method is based upon the equation:

$$\mathbf{Q = C*I*A}$$

Where:

Q = Peak flow rate (cfs)

I = rainfall intensity (inches/hr)

A = Area (acres)

Further discussion on determining the input parameters is discussed in the following sections

3.5.1 C-Coefficient Determination

The FCDMC Hydrology Manual (Volume 1) commonly used by large flood control agencies, was used to determine C coefficients most appropriate for the subject drainage areas. Based upon the FCDMC Table 3.2 the 10-year C coefficient is used to determine other return period C values. Refer to excerpts from the FCDMC manual for further details. Appendix E contains copies of the tables cited above as well as a summary of the actual C coefficients used in this analysis.

Rainfall intensity was derived by finding the intensity of rainfall that corresponded to the unique time of concentration for the drainage area. The time of concentration for the drainage basin was derived using the Papadakis-Kazan equation:

$$\mathbf{Tc = 11.4*L^{0.50}*K_b^{0.52}*S^{-0.31}*i^{-0.38}}$$

Tc = Time of concentration (hours)

L = Longest flow path length (miles)

K_b = Watershed resistance coefficient (Use Table 3.1)

S = Watercourse slope (ft/mile)

i = Rainfall intensity (inches/hour)

3.5.2 Rainfall Intensity Determination

Kb was derived from the *Drainage Design Manual for Maricopa County Arizona, Vol. 1,* Table 3.1. The K_b value is a function of drainage area and relative roughness. Since the drainage areas varied the Kb values were also different. For this drainage area a moderately rough watershed roughness (type B) was assumed for the lower alluvial fan areas and around washes. A moderately high roughness (type C) was assumed for the upper hilly portion of the watershed. For areas where both conditions exist, the predominant condition was taken for the whole. Where both conditions existed equally, the more conservative condition was chosen as representative of the area.

Since rainfall intensity in the Tc equation was unknown an iterative process was taken to determine the Tc and i. An initial guess for Tc was made using the FCDMC Figure 3.2. Rainfall Intensity Frequency relation and the equation are given as:

$$\mathbf{i = i_p * (P^6_{10})/2.07}$$

Where:

i_p = intensity for the Phoenix Metropolitan area

P^6_{10} = the 10-year, 6 hour precipitation depth for the point of interest.

Intensity was found for this duration. The intensity was then substituted into the Tc equation. If Tc was close to the initial Tc then the calculation was finished. If the new Tc was different then a new estimate for the rainfall intensity was made. This process was repeated until Tc and intensity converged. The results of the analysis are shown in the following table.

3.6 Hydrologic Results

HEC-1 results are contained in Appendix D. Rational Method calculations are contained in Appendix E. Table 2 summarizes the peak discharge values.

3.7 Indirect Method Verification

In order to establish confidence in the results of the computerized hydrologic analyses, it was important to develop some procedure to calibrate and/or verify the computer results with measured data. Normally the preferred approach is a two-step process, calibration followed by verification.

Calibration is the process of changing model coefficients, or other judgmental input parameters, until the model matches (with reasonable accuracy), the results from a measured event. Verification is the process of checking a calibrated model against a data set not utilized in the calibration process.

As might be expected, the scarcities of measured data made the calibration/verification process a difficult achievement. However, the absence of measured data can be overcome by employing several independent methodologies to calculate peak discharge values at the same concentration points used in the hydrologic analyses. These independent estimates can be compared to the results of the hydrologic analyses to see if sufficient discrepancies could warrant adjustments to the model input parameters. In the absence of measured rainfall/runoff data, the verification process can only be used as a guide to ensure that the model is not producing gross inaccuracies in the calculation of peak discharge values.

The Arizona Department of Transportation's (ADOT's*) Highway Drainage Design Manual - Hydrology* provides three indirect methods of checking the reasonableness of discharges calculated using analytical procedures:

- Method 1 - Unit peak discharge plots (plots of drainage area versus peak discharge/unit area)
- Method 2 – USGS data for Arizona
- Method 3 - Regional regression equations

Peak discharge values from regional regression equations are defined in the USGS. publication entitled *Methods for Estimating Magnitude and Frequency of Floods in the Southwestern United States* (summarized in the ADOT Hydrology Manual). Results of Methods 1-3 and peak discharge vs. drainage area relationships for the 25, 50 and 100-year event are found in Table 3. ADOT Figures for indirect methods, with computed primary peak discharge values superimposed on ADOT Figures, can be found in the Appendix F.

The results of the analysis are presented below. Values calculated appear to support the Hydrologic calculations flow rates.

Table 2: Hydrologic Results

CP	Basin A (Acres)	Basin A (Miles)	Method	Q5 (cfs)	Q10 (cfs)	Q25 (cfs)	Q100 (cfs)
1	9	0.01	Rational	25	28	38	55
2	970	1.52	HEC-1	213	236	418	832
3	150	0.23	Rational	200	240	360	560
4	36	0.06	Rational	39	46	71	110
5	272	0.42	HEC-1	67	124	225	446
6	237	0.37	HEC-1	139	230	387	652
7	51	0.08	Rational	110	140	190	250
8	1126	1.76	HEC-1	187	333	641	1272
9	290	0.45	HEC-1	92	164	292	559
10	214	0.33	HEC-1	69	126	224	414
11	119	0.19	Rational	140	170	250	400
12	1467	2.29	HEC-1	251	331	555	1030
13	1395	2.18	HEC-1	196	375	724	1475
14	145	0.23	Rational	150	180	280	440
15	821	1.28	HEC-1	110	174	296	659
16	158	0.25	Rational	150	180	280	450
17	65	0.10	Rational	75	91	140	220
18	1386	2.17	HEC-1	162	218	414	720
19	107	0.17	Rational	120	140	220	340
20	530	0.83	HEC-1	112	224	436	923
21	1413	2.21	HEC-1	159	218	417	728
22	122	0.19	Rational	120	140	220	340
23	184	0.29	HEC-1	54	93	164	314
24	1025	1.60	HEC-1	133	232	429	941

Table 3: Indirect Method Verification

4 HYDRAULIC ANALYSIS

The conceptual drainage plan is characterized as a pass through system. The proposed construction will not re-route or change existing flow paths significantly. Minor local changes may be necessitated; however, the predominant flow patterns will be maintained. Pipe culverts were selected to route off-site flows through the test track prism. It will be decided whether flow passing through the security fence needs to be conveyed by a pipe culvert system or simply an opening in the fence. The desired level of service has been tentatively set as the 25-year event for the off-site flows, and 5-10 year event for the on-site flows.

4.1 Culvert analysis

A preliminary culvert analysis will be carried out to determine initial culvert sizes. Pipe culverts will be analyzed using the Federal Highway Administration (FHWA) software HY-8 version 6.1. HW/D will be kept at 1.0 to1.5. HW/D is the ratio of headwater depth at the inlet to the diameter of the culvert. Figure 3 shows possible culvert locations as indicated by concentration point call outs (cp). Detailed mapping will be required to determine the location of culvert crossings. Additional crossings may be added to maintain vegetation corridors.

4.2 Ditches

Ditches may be used to collect some flows coming to the track prism. These flows will then be routed to their traditional outfall locations using existing or new ditches. Ditch sizing will be determined as the design progresses.

4.3 On-site

Further on-site hydrology and hydraulics will be addressed as the design progresses and matures. However, it is anticipated that storm water will be drained off the track surface to ditches. This water will be conveyed to detention basins or allowed to outfall to adjacent locations. The final design will show the ultimate configuration/plan.

5 CONCLUSIONS AND RECOMMENDATIONS

Premier Engineering Corporation was contracted to perform a drainage analysis for the proposed Hot Weather Test Complex (HWTC) at YPG. HEC-1 and Rational Method analyses were performed to quantify the off-site flows impacting the site. Table 2 in section 3 summarizes the design peak flow rates. These flow rates were checked against peak flow rates generated using regression equations found in ADOT's drainage manual. Verification method results supported the HEC-1 and Rational model results. Table 3 in section 3 presents the results of the verification analysis.

Figure 3 in section 3 shows the off-site flows.

Premier proposes the use of a pass through system to route stormwater through the HWTC prism. On-site flows will drain off of the test track into ditches, where it will be collected in retention basins, or allowed to drain to adjacent areas. A preliminary culvert analysis will be performed during the preparation of the final drainage report. A comprehensive hydraulic analysis will be conducted when updated planning and design information is available.

6 REFERENCES

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